A self-centering moment connection with steel-yielding hysteretic elements for earthquake-resistant high-performance steel frames

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Abstract

Self-centering steel moment resisting frames (SC-MRFs) have been developed as an alternative to conventional steel moment resisting frames (MRFs) aiming to eliminate structural damage and to minimise residual drifts under the design earthquake. In the present work, a new typology for self-centering beam-to-column connections is proposed. The connection makes use of post-tensioned high-strength strands which provide the required strength and self-centering capacity. Furthermore, a new energy-dissipation system (ED) which can be easily replaced after the design earthquake is introduced. The connection performance is experimentally validated under quasi-static cyclic loading conditions. The experimental results show that the proposed connection exhibits adequate self-centering behaviour and energy dissipation. The behaviour of the connection specimens is investigated up to failure, bringing the connections to drift levels far beyond the expected design levels, in order to identify the possible failure modes at ultimate collapse states.

Keywords: steel frames, seismic design, self-centering, hysteresis.

1 INTRODUCTION

Conventional ductile steel moment-resisting frames (MRFs) are currently designed to form a global plastic mechanism under strong earthquakes through the development of plastic hinges at the end of beams and at the base of the columns (Eurocode 8 2004). This design approach provides well known advantages such as acceptable behaviour able to protect human life, economy, low base shear force and controlled total floor accelerations. However, plastic hinges in structural members involve significant cyclic inelastic deformations and local buckling which result in difficult to inspect and repair damage as well as residual drifts. Therefore, conventional steel MRFs result in socio-economical losses such as damage repair costs and loss of building use or occupation after a major seismic event. In addition, they may result in building demolition due to the complications associated with repairing large residual drifts. Research efforts developed self-centering steel moment resisting frames (SC-MRFs) with post-tensioned (PT) connections (Garlock et al. 2007). SC-MRFs have the potential to eliminate inelastic deformations and residual drifts under strong earthquakes as the result of several features: softening force-drift behaviour due to separations (gap openings) developed in beam-to-column connections; re-centering capability due to elastic pre-tensioning elements (e.g., high strength steel bars) providing clamping forces to connect beam and columns; and energy dissipation capacity due to energy dissipation elements (EDs) which are activated when gaps open. When properly designed, these EDs can be easily inspected and replaced.

2 SC PT CONNECTION WITH WHP

The proposed SC PT connection was designed to provide stable self-centering capability and increased energy dissipation with the aid of high strength steel bars and WHPs which do not interfere with the floor slab and are very easy to replace. Fig. 1 shows a 3D representation of the proposed connection. The figure represents the actual configuration of an exterior connection of a steel SC-MRF. Two high strength steel bars located at the mid-depth of the beam, one at each side of the web, are anchored to the exterior column flange and pass through holes drilled on the column flanges. The bars are post-tensioned and hence, clamp the beam to the column while providing self-centering capability. WHPs consist of two pairs of steel pins which are inserted in aligned holes drilled on the web of the beam and on strong supporting plates. The pairs of WHPs are symmetrically placed (close to the top and bottom beam flange) to provide increased lever arm and hence, increased internal moment resistance under cyclic connection rotations.

As shown in Fig. 2, WHPs are designed to have an hourglass shape to provide enhanced energy dissipation and fracture capacity (Kobori et al. 1992). Both sides of the beam web are reinforced with steel plates which increase the contact surface between the WHPs and the web. In this way, possible ovalisation of the sides of the holes drilled on the web and the reinforcing plates under the high WHP bearing forces will be negligible and connection pinching behaviour can be avoided. The strong supporting plates are welded on the column flange. The connection also includes beam flange reinforcing plates to avoid excessive early yielding in the beam flanges under the high PT bars forces. In addition, the panel zone is strengthened with horizontal stiffeners and continuity plates along the web of the column.

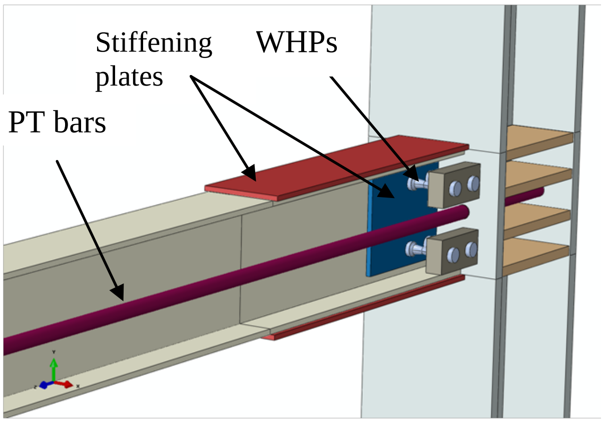


Figure 1. 3D representation of the proposed self-centering PT connection

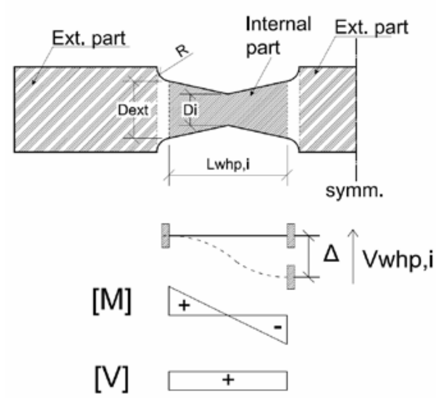


Figure 2. Shape, assumed static system and internal force diagrams for the WHPs.

3 DESIGN OF PROTOTYPE SC PT CONNECTION WITH WHP

The design focuses on the exterior connection of the 2nd floor of a prototype steel building. This connection is designed as a self-centering PT connection with WHPs using the methodology presented by Garlock et al. (2007). This methodology uses two major performance objectives and associated structural limit states, namely: (1) Immediate Occupancy (IO) under the design basis earthquake (DBE) by avoiding damage in beams and columns while permitting gap opening (it is assumed that damaged WHPs can be replaced without disturbing building occupation); and (2) Collapse Prevention (CP) under the maximum considered earthquake (MCE) by avoiding PT bar yielding and beam local buckling while permitting minor yielding in beams and columns. The MCE has intensity equal to 150% the intensity of the DBE.

The above performance objectives proposed by Garlock et al. (2007) may result in sudden loss of strength and stiffness due to undesirable failures related to beam local buckling or tendon yielding when the connection is deformed beyond the MCE drift limit. In this work, the details proposed by Kim & Christopoulos (2009a) are employed to avoid local buckling and form a ductile plastic hinge at the end of the flange reinforcing plate for drifts equal or higher than the MCE drifts. This design approach is slightly different than the one presented in Kim and Christopoulos (2009b) where the ductile plastic hinge is formed for drifts equal or higher than the DBE drifts.

4 WHP COMPONENT TESTING

Component tests on the WHPs were conducted in order to assess their energy dissipation capacity and ductility under cyclic loading. Fig. 3 shows the experimental setup for the component testing. Two pins were tested in order to reproduce the behaviour of the WHPs in the proposed SC connection typology. The supporting plates were welded on a fixed plate and the plate simulating the web of the beam was attached to the actuator applying the load. The thicknesses of the plates were equal to the thicknesses of the corresponding components in the connection tests described later.

The induced displacement history was chosen to be consistent with that used for the connection tests, i.e., the required displacement of the pins due to the imposed connection rotation in the actual tests was calculated and applied as the component testing displacement history.

The WHPs were designed in order to guarantee a stable and ductile hysteretic behaviour and to maximise the energy dissipation capacity during a seismic event. The hourglass geometry of the pins was calibrated in order to be consistent with the bending-moment diagram, as shown in Fig. 2. In this way, a more efficient energy dissipation capability is ensured. Moreover, probable associated pinching effects are avoided. Fig. 4a shows the WHPs at ultimate deformation levels during the component tests. This deformation corresponds to a connection rotation of about 0.06 rad. The pins were capable of sustaining repeated large inelastic cycles without fracturing prior to an imposed displacement corresponding to 7% drift. Fig 4b plots the force-displacement cyclic relationship achieved by the WHPs. The pins provided a force equal to 160kN. It is evident that the developed geometry resulted in a highly-dissipative system with good hysteretic behaviour and high ductility levels.

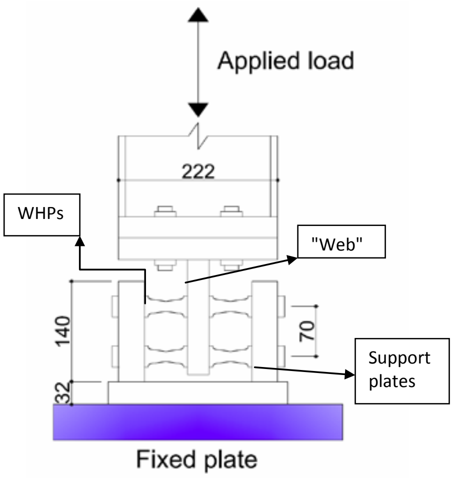
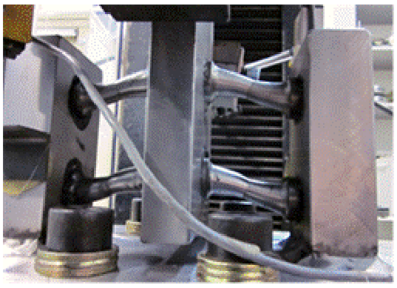
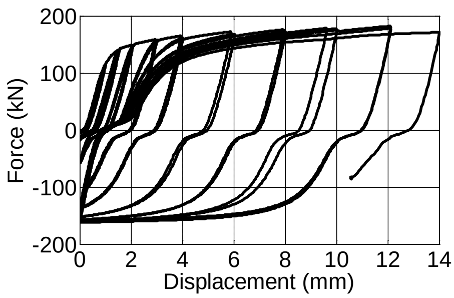


Figure 3. The test setup for the component tests





a) b)

Figure 4. a) Ultimate deformation and b) hysteretic force-displacement loop of the WHPs.

5 LARGE-SCALE CONNECTION TESTS

The test setup is schematically shown in Fig. 6. The 250UB37 beam was connected to a strong 310UC158 column. Two additional steel members were welded to the column to form a truss system in order to minimise the column deformations (the 310UC158 horizontal member and the 200UC52 diagonal member). The whole system was bolted on the strong floor. The imposed displacement history was applied vertically by a 2000kN-capacity and 250mm-stroke hydraulic actuator. The actuator was positioned at a distance of 1800mm from the beam-column face and thus was able to impose a joint rotation equal to 7%, much larger than the target rotation of 2.4%, corresponding to the MCE level. Two specimens representing the proposed SC post-tensioned connection were designed according to the procedure described in Section 3. Due to laboratory limitations, the specimens were designed at a 0.6 scale with respect to the original dimensions of the prototype building. The resulting beam and column sizes and relevant details of the connections are summarised in Table 1. The two specimens were identical except that four longitudinal stiffeners were welded on the web and four 27mm-holes were drilled on the flanges of the second specimen, according to Kim & Christopoulos (2009a). The details of this configuration are shown in Fig. 6. The purpose of this configuration is to eliminate local buckling effects on the beam web or flanges. Instead, a plastic hinge is intended to be formed at the end of the reinforcing plate. In this way, sudden failures and local instabilities are avoided and a ductile ultimate failure mode under large inelastic drifts is ensured.

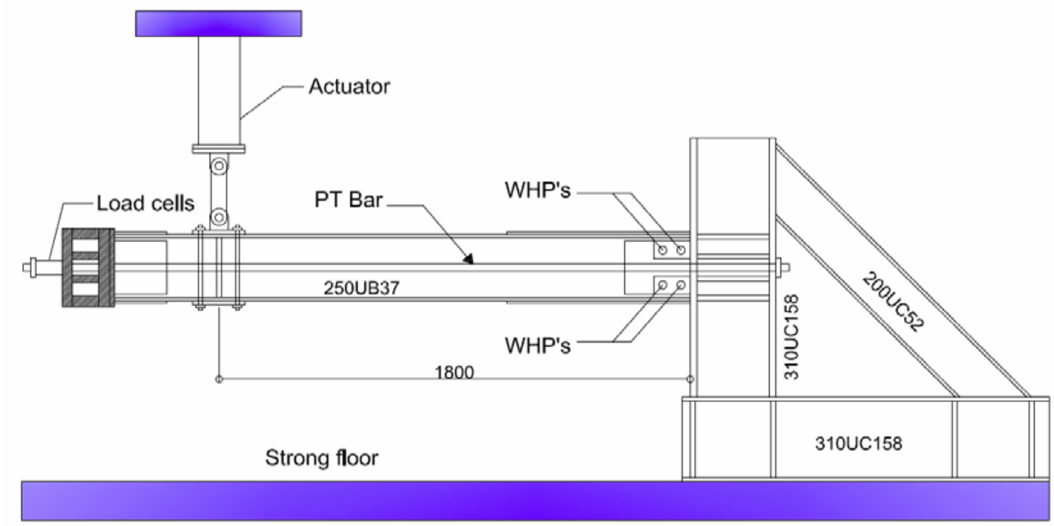


Figure 5. The test setup

Table 1. Specimen details

Specimen:	SC-WHP1	SC-WHP2
Beam	250UB37	250UB37
Column	310UC158	310UC158
$F_{PT,i}$ (kN)	518	504
L_{rp}^{*} (mm)	700	700
Long. stiffeners	NO	YES

$F_{PT,i}$: Initial post-tensioning force, Lrp: Length of reinforcing plates on the beam

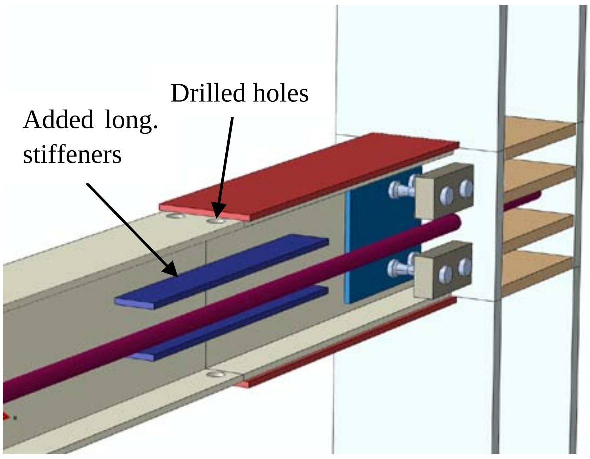


Figure 6. Details of the second specimen

6 EXPERIMENTAL RESULTS

The specimens were loaded under displacement-controlled conditions, according to the AISC loading protocol (AISC 2005). The moment-rotation hystereses for the two specimens are shown in Figs. 7 and 8. It is shown that the specimens had a full self-centering capacity up to rotations equal to 0.05 rad. In fact, no damage in any part except the WHPs was observed up to this rotation levels. This rotation is considerably larger than the specified target rotation of 0.035 rad, which is set as the required rotation capacity of a connection for Ductility Class High (DCH) structural design concept in Eurocode 8. The connections were loaded to excessive drift levels in order to identify all possible failure modes. A final cycle of 10% drift was imposed for this purpose. Figs. 7 and 8 show that after the last large imposed displacement the connection lost its self-centering capability and a permanent drift was present at the end (indicated as "ultimate response" in the diagrams).

To assess the ability of the proposed connection to be easily repaired after a strong earthquake, the SC-WHP1 specimen was imposed to displacements up to the DBE level (i.e. 1.6% drift). The test was then interrupted and the damaged WHPs were substituted by a set of new ones. After the substitution of the WHPs, the test was restarted. Fig. 7 plots superimposed loops from the tests before and after the replacement of WHPs (named "1st run" and "2nd run" in the graph). During the first cycles, the connection loses part of its initial strength and stiffness as a result of the unavoidable local yielding of the beam edges due to the bearing on the column as the gap opens and closes. Nevertheless, Fig. 7 shows that the connection gains its strength and stiffness after rotations of 0.005rad. In fact, the two loops are perfectly superimposed to each other, demonstrating that the proposed connection can be very easily repaired after a strong earthquake without compromising its stiffness, strength and energy dissipation capacity.

The failure modes were different in the two specimens. The failure mode of specimen SC-WHP1 was the local buckling of the beam which took place immediately after the end of the reinforcing plates on the beam flanges (Fig. 9). The buckling initiated at the web and propagated to the flanges. This failure caused a significant drop in the stiffness of the connection as shown in Fig. 7. The detailing employed for specimen SC-WHP2 resulted in a different failure mode. No web or flange buckling was observed in the beam section after the end of the beam flange reinforcing plates. In addition, no plastic hinge was developed in this region, indicating that the design procedure employed was rather conservative. Fig. 10 shows the failure mode of this specimen, i.e., excessive yielding in the beam flanges at the beam-column interface due to large bearing forces. Local yielding at the bearing face was more pronounced than in specimen SC-WHP1 and resulted in a gradual reduction in post-tensioning force. A sudden reduction of 15kN in the force of the PT bars was observed as a result of the local failure at the final loading cycle.

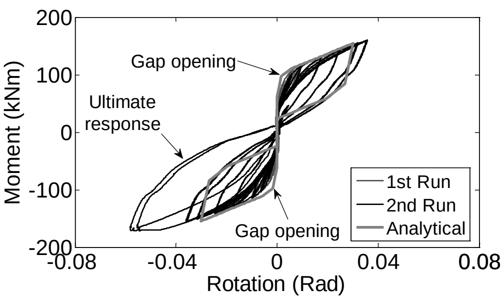


Figure 7. Moment-rotation hysteresis loop for specimen SC-WHP1

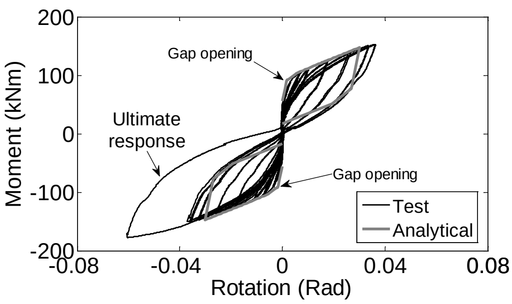


Figure 8. Moment-rotation hysteresis loop for specimen SC-WHP2.

7 SUMMARY AND CONCLUSIONS

The main conclusions of the large-scale experimental tests can be summarised in the following:

- The proposed post-tensioned connection showed stable and predictable self-centering behavior with negligible residual drifts under the design earthquake, while the initial stiffness and energy dissipation capacity are comparable to those of a welded fully-restrained MRF.

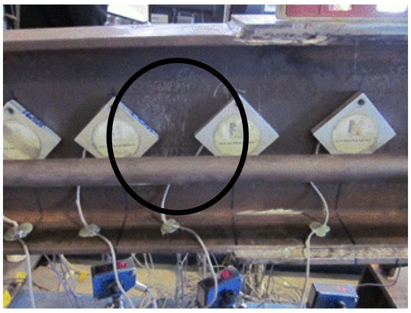


Figure 9. Local buckling of beam web at the end of the reinforcing plate (specimen SC-WHP1)

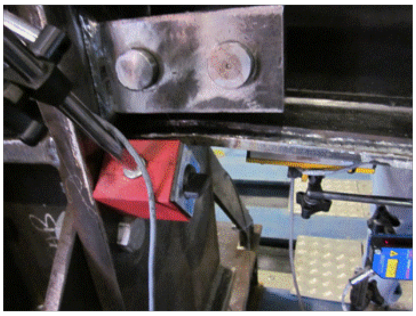


Figure 10. Local buckling of beam web at the end of the reinforcing plate (specimen SC-WHP1)

- Story drifts up to 6% and connection rotations up to 0.035rad were achieved with self-centering behavior, while the damage was mainly concentrated in the WHPs.
- Repeated tests on one specimen demonstrated that the connection can be easily repaired by replacing the WHPs. Moreover, the replacing process does not involve any additional bolting or welding in the connection region.
- By introducing appropriate detailing local buckling at the region after the reinforcing plates is avoided. However, special measures should be taken to eliminate local yielding at the beam-column bearing interface due to high compressive forces.
- There is a need for extensive research which will provide with an accurate and reliable design methodology for post-tensioned steel connections and could be safely adopted in the design practice of self-centering steel moment-resisting frames.

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